

# **SEISMIC EVALUATION AND RETROFIT OF A RC FRAME STRUCTURE**

**A THESIS SUBMITTED IN PARTIAL FULFILLMENT  
OF THE REQUIREMENTS FOR THE DEGREE OF**

**Bachelor of Technology**

**In  
CIVIL ENGINEERING**

**By  
PRAVAL PRIYARANJAN  
(108CE028)**



**DEPARTMENT OF CIVIL ENGINEERING  
NATIONAL INSTITUTE OF TECHNOLOGY, ROURKELA  
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**UNDER THE GUIDANCE OF**

**PROF. P. SARKAR**



**DEPARTMENT OF CIVIL ENGINEERING**

**NATIONAL INSTITUTE OF TECHNOLOGY, ROURKELA**

**May, 2012**



**NATIONAL INSTITUTE OF TECHNOLOGY**  
**ROURKELA**  
**CERTIFICATE**

This is to certify that the thesis entitled “Seismic Evaluation and Retrofit of a RC frame structure” submitted by Mr Praval Priyaranjan in partial fulfilment of the requirements for the degree of Bachelor of Technology in Civil engineering at National Institute of Technology, Rourkela (Deemed University) is an authentic work carried out by him under my supervision and guidance.

To the best of my knowledge, the matter embodied in the thesis has not been submitted to any other university/institute for the award of any degree or diploma.

Date:

Prof. P. Sarkar  
Department of Civil Engineering  
National Institute Of Technology  
Rourkela-769008

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B.TECH

DEPARTMENT OF CIVIL ENGINEERING

N.I.T ROURKELA

## **ABSTRACT**

Recent earthquakes in India show that not only non-engineered but also engineered buildings in our country are susceptible even to moderate earthquakes. Indian Standard IS 1893 is revised in 2002. A number of buildings those were designed as per the previous code may not comply with the present code. Therefore evaluating seismic performance of a building and proposing suitable retrofit measure is an important area of study in this context. In the present study an attempt has been made to evaluate an existing building located in Guwahati (seismic zone V) using equivalent static analysis. Indian Standard IS-1893:2002 (Part-1) is followed for the equivalent static analysis procedure. Building is modeled in commercial software STAAD Pro. Seismic force demand for each individual member is calculated for the design base shear as required by IS-1893:2002. Corresponding member capacity is calculated as per Indian Standard IS456:2000. Deficient members are identified through demand-to-capacity ratio. A number of beams and column elements in the first floor of the present building are found to be deficient that needs retrofitting. A local retrofitting strategy is adopted to upgrade the capacity of the deficient members. This study shows that steel jacketing is an efficient way to retrofit RC members to improve flexure as well as shear capacity.

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## 1. INTRODUCTION

### 1.1 Objectives:

To perform seismic evaluation of a residential building in Guwahati, Assam and provide methods for retrofitting of members in case the members fail under the load combinations prescribed in IS 1893-2002.

### 1.2 Methodology:

- a) The methodology adopted to perform the seismic evaluation of the building requires an understanding of equivalent lateral force procedure also recognized as equivalent static procedure in literature.
- b) An in depth knowledge of STAADPro software is required as the building was modeled in STAADPro and post analysis data obtained from it was used in the analysis of the structure.
- c) The demand to capacity ratio of members was calculated to analyze the seismic stability of the structure under the various load combinations in accordance with IS 1893-2002 (part 1)
- d) Suitable retrofit measures were proposed for beams and columns failing in shear and flexure

### 1.3 Scope of the study:

In the equivalent static procedure of seismic analysis, the seismic loads are applied to the centre of mass of the storey, but in STAADPro I have assumed the seismic loads to be nodal loads and applied it to nodes dividing the total lateral storey loads in equal proportion per node and not at

the exact centre of mass of the storey. While considering retrofit measures for the structure, analysis of structure post concrete jacketing was kept outside the scope of this study and only flexural analysis of members post steel plating was taken up. It was assumed that there would be sufficient adhesion between plates and concrete so that there is no failure due to bonding.

#### 1.4 Basics of Seismic evaluation:

##### **Response spectra:**

Interaction between ground accelerations and structural systems are reported through response spectrum. Plots peak responses over time for a range of single-degree-of-freedom (SDOF) systems subjected to a particular base motion as a function of their natural frequency  $\omega_i$ , or vibration period  $T_i$ . The resulting plot can then be used to pick off the response of any linear system, given its natural frequency of oscillation. Response spectra are used by earthquake engineers for analyzing the performance of structures in earthquakes, since many behave principally as single degree of freedom systems. The purpose of the response spectrum is to know the response of a single degree of freedom system if the ground moves as per the given accelerogram. An accelerogram is the recording of the acceleration of the ground during an earthquake. Response may mean any quantity like acceleration, velocity or deformation<sup>[1,2]</sup>.

The figure below shows the accelerogram for the earthquake that hit the El-Centro city in Imperial valley of California.

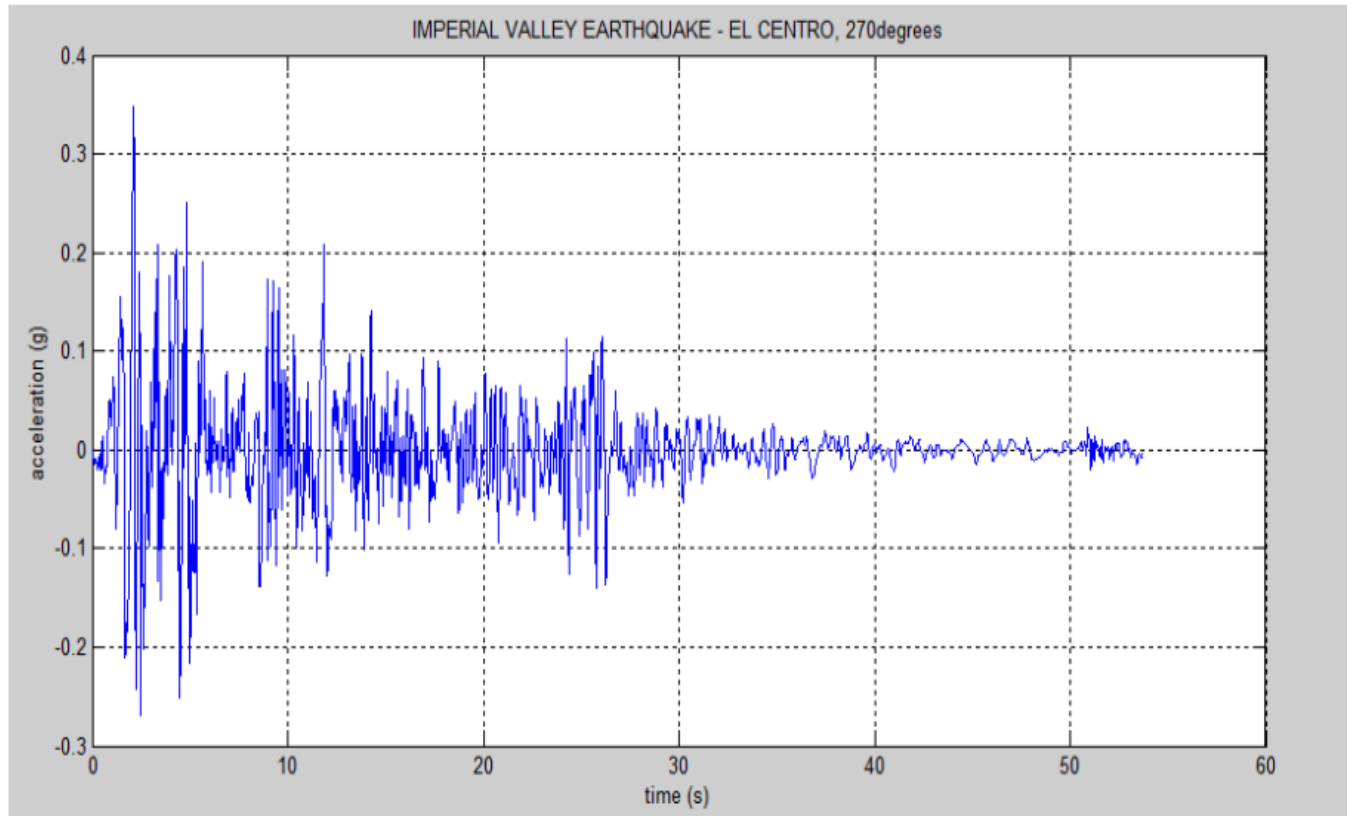


Figure 1: Accelerogram for the earthquake affecting El-Centro city in 1940 <sup>[3]</sup>

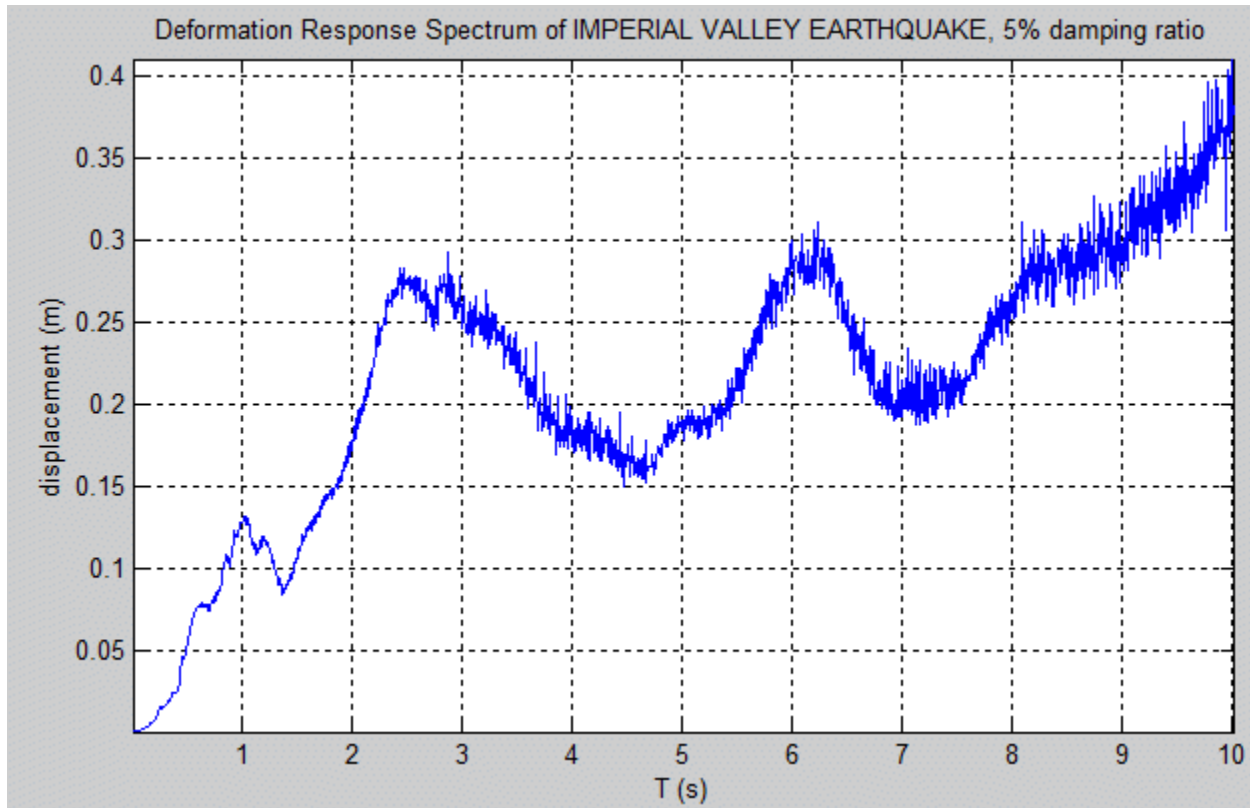


Figure 2: Deformation Response Spectrum for 5% damping ratio <sup>[3]</sup>

### Design spectrum:

Response spectrums vary a lot even with a little change in natural frequency of the structure and so have very irregular shape with local maxima and minima. For design purposes, local maxima and minima are ignored because natural period of structures cannot be calculated very accurately. Thus design spectrum is a smooth response spectrum specifying level of seismic resistance required for design. It is a specification of the required strength of the structure.

The strength depends on the following factors:

- Frequency
- Maximum velocity
- Maximum displacement
- Maximum acceleration

Design spectrum must also be accompanied by:

- Load factors, as different choices of load factors will lead to different seismic safety of the structure.
- Damping, variations in the values of damping used in the design will affect the design force.
- Method of calculation of natural period of the structures, which depends upon the assumptions made while modeling.<sup>[2,4]</sup>

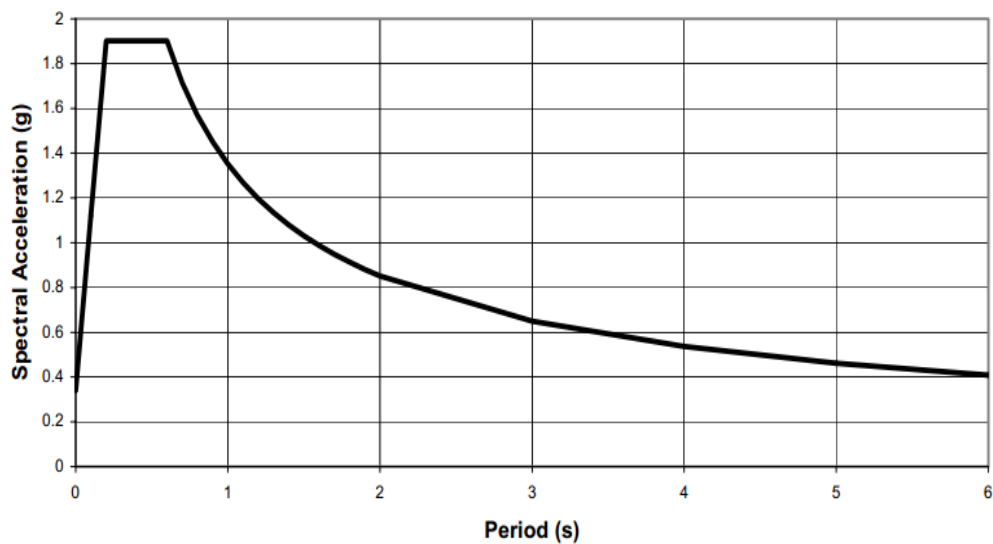


Figure 3: A typical design spectrum<sup>[5]</sup>

## 2. SEISMIC EVALUATION METHODS

### Seismic evaluation methods:

- Preliminary investigation
- Detailed evaluation

#### 2.1 Preliminary investigation:

The preliminary evaluation is a quick procedure to establish actual structural layout and assess its characteristics that can affect its seismic vulnerability. It is an approximate method based on conservative parameters to identify the potential earthquake risk of a building and can be used for screening of buildings for detailed evaluation. It also helps the design engineers to get acquainted with the building, its potential deficiencies and behavior. A site visit is done as a part of preliminary investigation in order to familiarize with the building and take note of the ground conditions which are not reported in the drawings.<sup>[6]</sup>

#### 2.2 Detailed evaluation methods:

- Equivalent static method
- Response spectrum method

##### 2.2.1 **Response spectrum method:**

Response spectrum analysis is a procedure for computing the statistical maximum response of a structure to a base excitation. Each of the vibration modes that are considered may be assumed to respond independently as a single-degree-of-freedom system. Design codes specify response

spectra which determine the base acceleration applied to each mode according to its period (the number of seconds required for a cycle of vibration).

For example: the diagram below shows the Basic Seismic Hazard Acceleration Coefficient specified in NZS 4203 for deep soil sites. Each curve represents a different ductility factor. The design response spectrum is obtained by multiplying these curves by a structural performance factor, a risk factor, a zone factor, and limit state factor.

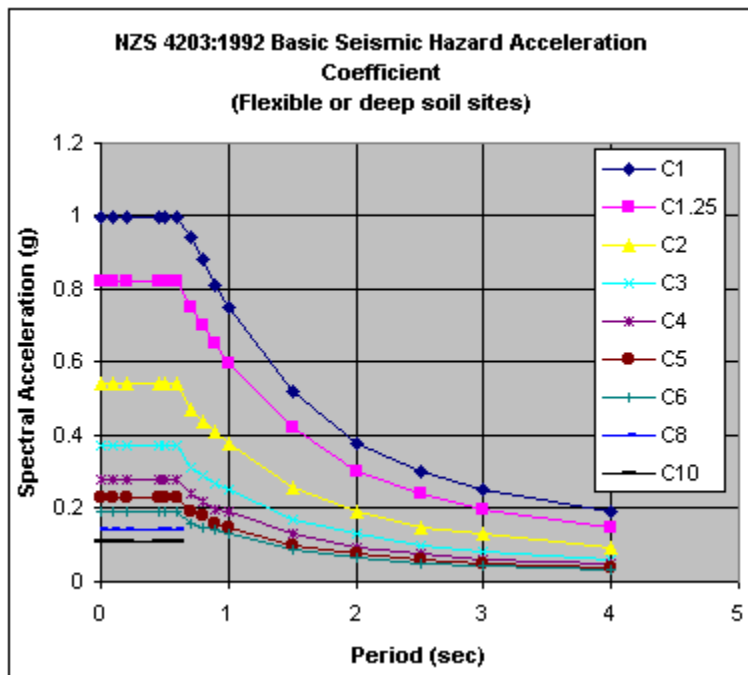


Figure 4: Basic seismic hazard coefficient specified in NZS 4203<sup>[7]</sup>

Having determined the response of each vibration mode to the excitation, it is necessary to obtain the response of the structure by combining the effects of each vibration mode because the maximum response of each mode will not necessarily occur at the same instant, the statistical maximum response, where damping is zero, is taken as the square root of the sum of the squares (SRSS) of the individual responses.



Response spectrum analysis produces a set of results for each earthquake load case which is really in the nature of an envelope. It is apparent from the calculation, that all results will be absolute values - they are all positive. Each value represents the maximum absolute value of displacement, moment, shear, etc. that is likely to occur during the event which corresponds to the input response spectrum.<sup>[7]</sup>

To explain the response spectrum concept, we consider a SDOF system in which an external action is applied like an applied force or support displacement. For the response spectrum, it is necessary to evaluate the value of the maximum response, which may be determined once the equation  $q(t)$  is fully known.

The equation of displacement  $q(t)$  for a SDOF system with damping  $\xi$  and natural frequency  $w$  is

$$\ddot{q}(t) + 2w\xi\dot{q}(t) + w^2q(t) = \frac{p(t)}{m} \quad (2.1)$$

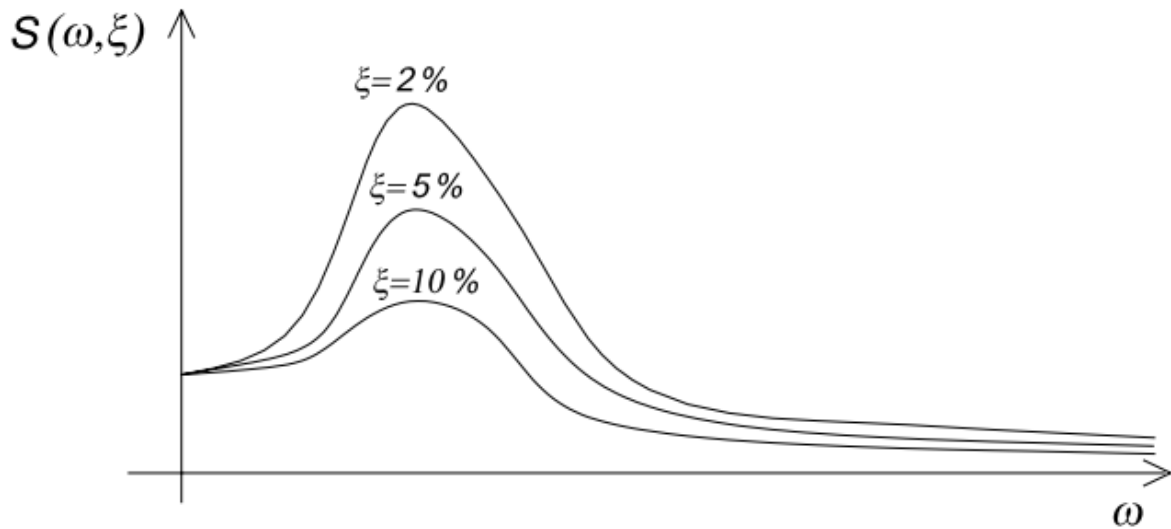


Figure 5: Typical representation of response spectrum<sup>[8]</sup>

Response spectrum analysis applied to MDOF systems<sup>[8]</sup> :

For direction J the maximum value of modal coordinates in terms of displacements,  $Y'_{n,max}$  may be achieved if the displacement response spectrum  $S_d(w,\xi)$  is known.  $Y'_{n,max}$  is established from the response spectrum for the SDOF system with both the same natural vibration frequency,  $w_n$  and critical damping ratio  $\xi_n$

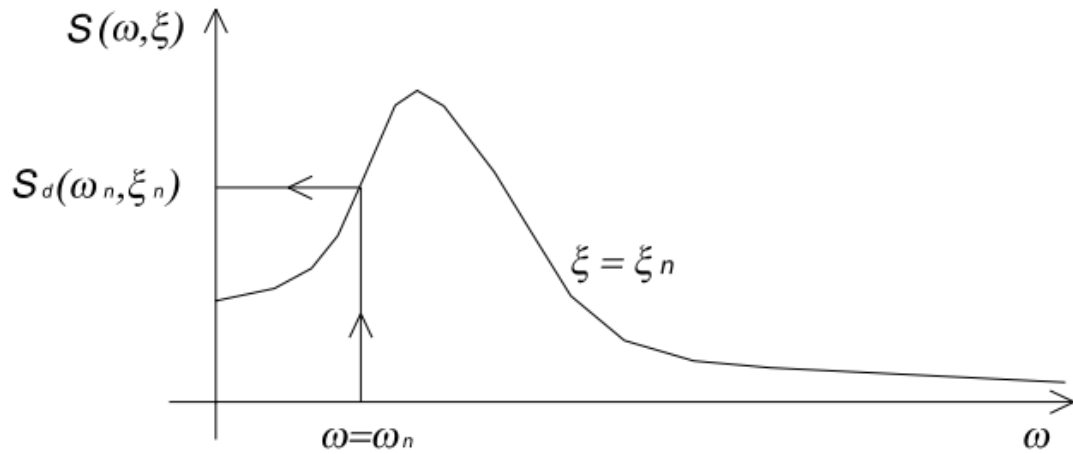


Figure 6: Explaining the response spectrum analysis procedure<sup>[8]</sup>

After establishing the maximum value of modal coordinate  $Y'_{n,max} = S_d$ , the modal participation factor is written as:

$$Y'_{n,max} = P_{ij}.S_{d,j}(w_n, \xi_n) \quad (2.2)$$

In the same manner we can calculate the maximum response in terms of acceleration,  $\ddot{Y}'_{n,max}$

$$\ddot{Y}'_{n,max} = P_{ij}.S_{a,j}(w_n, \xi_n) \quad (2.3)$$

Or velocity  $\dot{Y}_{n,max}$

$$\dot{Y}_{n,max} = P_{ij}.Sv,j(w_n, \xi_n) \quad (2.4)$$

( $P_{ij}$  is the modal participation factor for  $i$  th mode for direction  $J$ . it only depends upon the vibration mode shape, the mass distribution and the direction of each degree of freedom)

There is a need to combine the modal responses because the result provided are a set of extreme values which do not correspond to an equilibrium state and do not take place at the same time.

There are several methods to do it one of which is Square root of sum of squares method (SRSS) method.

In this method, the maximum response in terms of a given parameter,  $G$  (displacement, acceleration, velocity) may be estimated through the square root of the sum of the  $m$  modal response squares, contributing to the global response:

$$G \approx \sum_{n=1}^m (G_n)^2 \quad (2.5)$$

### 2.2.2 Equivalent static method:

The equivalent static method is the simplest method of analysis because the forces depend on the code based fundamental period of structures with some empirical modifiers. The design base shear is to be computed as whole, then it is distributed along the height of the building based on some simple formulae appropriate for buildings with regular distribution of mass and stiffness.

The design lateral force obtained at each floor shall then be distributed to individual lateral load resisting elements depending upon the floor diaphragm action.

Following are the major steps in determining the seismic forces:

### **Determination of base shear:**

For the determination of seismic forces, the country is classified in four seismic zones:

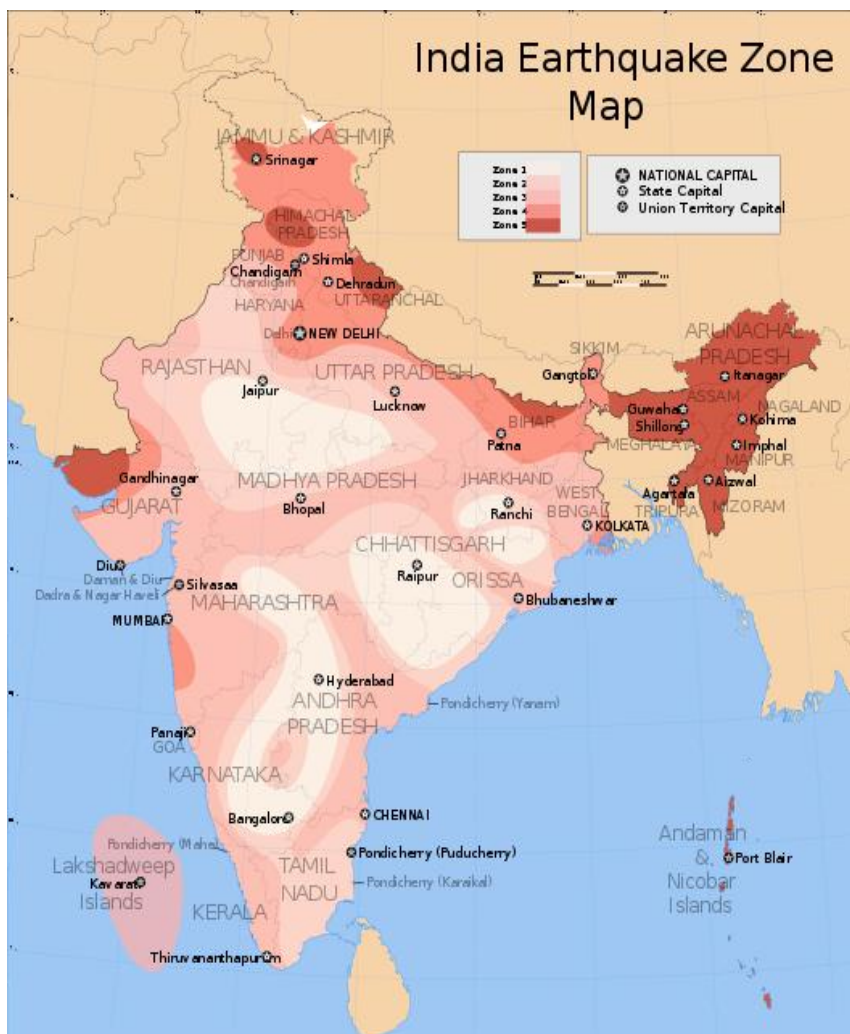


Figure 7: Showing seismic zones in India

The total design lateral force or design base shear along any principal direction shall be determined by this expression:

$$V_b = A_h W \quad (2.6)$$

Where,

$A_h$  = design horizontal seismic coefficient for a structure

$W$  = seismic weight of building

The design horizontal seismic coefficient for a structure  $A_h$  is given by:

$$A_h = (ZIS_a)/2Rg \quad (2.7)$$

$Z$  is the zone factor given in Table 2 of IS 1893:2002 (part 1) for the maximum considered earthquake (MCE) and service life of a structure in a zone. The factor 2 is to reduce the MCE to the factor for design base earthquake (DBE)

$I$  is the importance factor, depending upon the functional use of the structure, characterized by hazardous consequences of its failure, post-earthquake functional needs, historical or economic importance. The minimum values of importance factor are given in table 6 of IS 1893:2002

$R$  is the response reduction factor, depending on the perceived seismic damage performance of the structure, characterized by ductile or brittle deformations. The need for introducing  $R$  in base shear formula is an attempt to consider the structure's inelastic characteristics in linear analysis

as it is undesirable as well as uneconomical to design a structure on the basis that it will remain in elastic range for all major earthquakes. Note: IS code recommends that the value of  $I/R$  should not exceed 1.0 the values of  $R$  are given in Table 7 of IS 1893:2002 (part 1).

$S_a/g$  is the average response acceleration coefficient for rock and soil sites as given in figure 2 of IS 1893:2002 (part 1). The values are given for 5 % of damping of the structure.

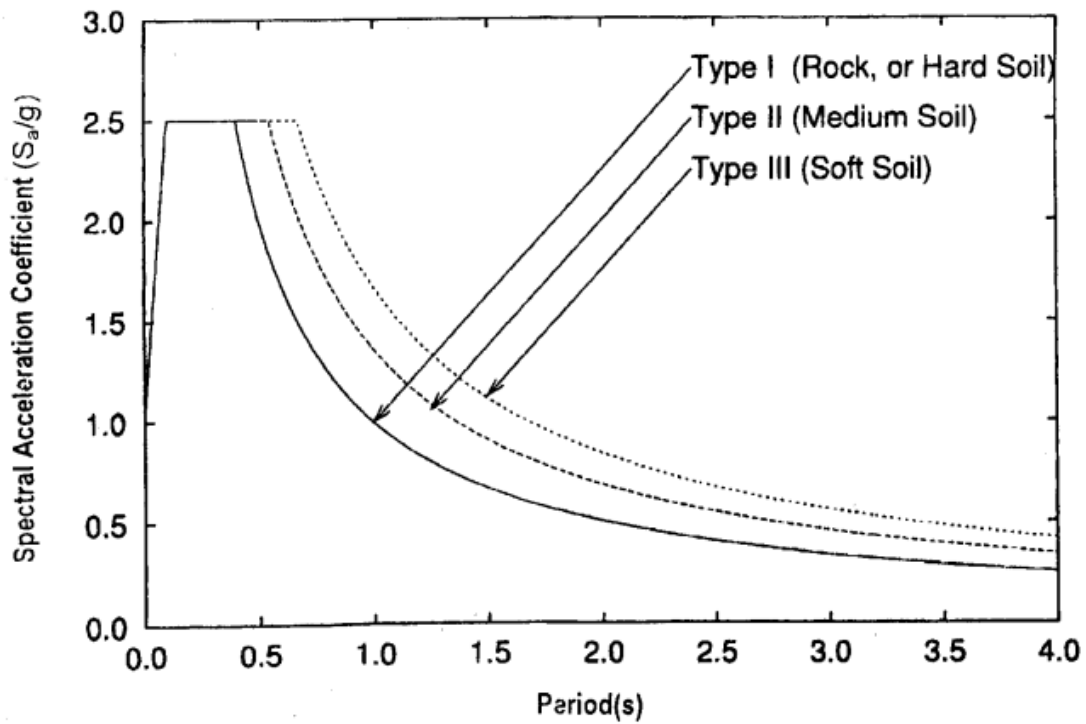


Figure 8: Response spectra for rock and soil sites for 5% damping [fig 2 of IS 1893:2002]

T, the fundamental natural period for buildings are calculated as per Clause 7.6 of IS 1893:2002 (part 1).

$$T_a = 0.075h^{0.75} \text{ for moment resisting frame without brick infill walls}$$

$$T_a = 0.085h^{0.75} \text{ for resisting steel frame building without brick infill walls}$$

$$T_a = 0.09 \frac{h}{\sqrt{d}} \text{ for all other buildings including moment resisting RC frames}$$

h is the height of the building in m and d is the base dimension of building at plinth level in m

#### **Lateral distribution of base shear:**

After the total design base shear is calculated, it is distributed along the height of the building. The base shear at any floor or level depends on the mass of the level and deformed shape of the structure. Earthquake forces can deflect a building into a number of shapes, the natural mode shapes of the building which in turn depend upon the degree of freedom of the building. A building can have infinite degree of freedoms but we convert it to finite degree of freedom by idealizing a multi storeyed building into a lumped mass model by assuming the mass of the building lumped at each floor level with one degree of freedom in the direction of lateral displacement in which the structure is being analyzed per floor, resulting in degrees of freedom equal to the number of floors.<sup>[4]</sup>

The magnitude of the lateral force at a floor (node) depends on:

- Mass of that floor
- Distribution of stiffness over the height of structure
- Nodal displacements in a given mode

IS 1893:2002 (part 1) uses a parabolic distribution of lateral force along the height of the building. Distribution of base shear along the height is done according to this equation:

$$Q_i = \frac{w_i h_i^2}{\sum_{j=1}^n w_j h_j^2} \quad (2.8)$$

Where:

$Q_i$  = design lateral force at floor  $i$

$W_i$  = seismic weight of floor  $i$

$h_i$  = height of floor  $i$  measured from foundation

$n$  = number of stories in the building or the number of levels at which masses are located



### 3. Building Analysis

#### 3.1 About the building:

Table 1: Building description:

<b>Building type</b>	<b>Reinforced concrete frame</b>
<b>Usage</b>	Residential apartment
<b>Location</b>	Guwahati, Assam
<b>Year of construction</b>	1999
<b>Number of stories</b>	Open ground + 4
<b>Plan dimensions</b>	25.2 m X13.95 m
<b>Building height</b>	15 m

Table 2: Grade of Materials

<b>Concrete</b>	<b>M 15</b>
<b>Reinforcing Steel</b>	Fe 415

### 3.2 Modeling in Staad Pro:

- Nomenclature:

I adopted a scientific approach to modeling in STAAD. In my approach I did not use any shortcut commands and worked only through the Staad editor. The most important part of modeling was the nomenclature of nodes, beams and columns. A proper nomenclature of nodes, beams and columns is very important as it gives you the exact idea where that member is located in the entire structure and has an added advantage while debugging. The nodes were named by giving their x, z co-ordinates a specific number and the y coordinate (along the height) was according to the floor number.

For example:

Node no: 15010 – is a level 1 node (first no) with x co-ordinate attributed to no. (50) and z co-ordinate attributed to no. (10)

From the figure below it can be seen that just by the node number we can know the exact position of any node.

Here all (10) nodes – 111(10) to 150(10) denote that they have the same z co-ordinate (1275 mm) and are of the first level (1).

Table 3: showing process of nomenclature of nodes in X direction

X in metre	Allotted no.	X in metre	Allotted no.
0	10	24.875	50
0.325	11	25.2	51
0.925	12	3.55	15
3.425	14	21.65	44
6.475	18	9.4	21
6.9	20	15.8	35
10.275	22	5.1625	17
10.925	23	20.0375	42
12.6	30	23.225	46
14.275	33	1.975	13
14.925	34	11.075	31
18.3	40	14.125	32
18.725	41	4.925	16
21.775	45	20.275	43
24.275	47		

Table 4 : showing process of nomenclature of nodes in Z direction

Z in metre	Alloted no.
-1.275	10
-3.525	12
-4.4	20
-6.225	22
-8	30
-9.825	31
-10.4	40
-11.875	42
-12.675	50
-3.275	11
-4.525	21
-10.9375	41

Now for example a node 15010 will have x co-ordinate (50) value (from table 3) = 24.875m, z coordinate (10) (from table 4) value = 1.275 m, which is verified from figure 11

Example 2: if a first level node would have existed having 3.425 m X coordinate and 8 m Z coordinate, its nomenclature would have been: (1),(14),(30) i.e. 11430

Similarly the nomenclature of beams and columns are done which gives us all information about the actual location of the member in the structure and not just any random number allotted by STAAD Pro.

- The supports were taken as all fixed.

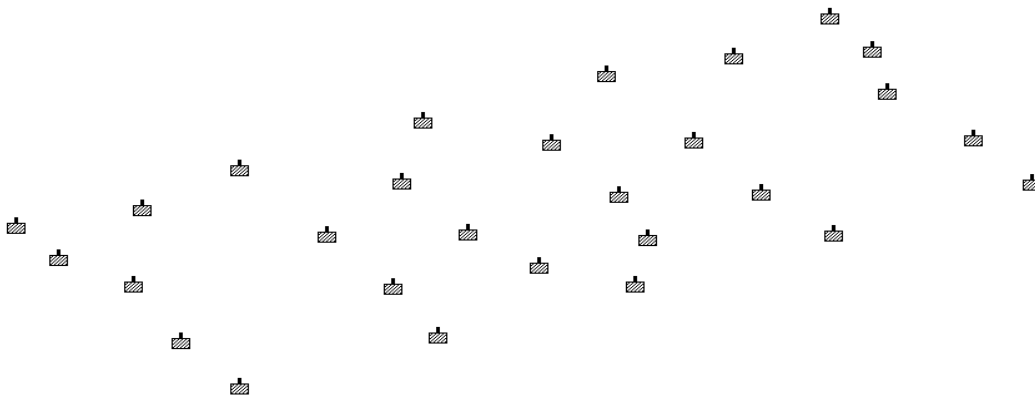


Figure 10: Showing fixed supports at foundation level

- Member properties were provided to beams and columns as provided in the plan of the structure.

Example:

11101 PRIS YD 0.5 ZD 0.3

11102 PRIS YD 0.5 ZD 0.3

Beams having depth 500 mm and 300 mm width

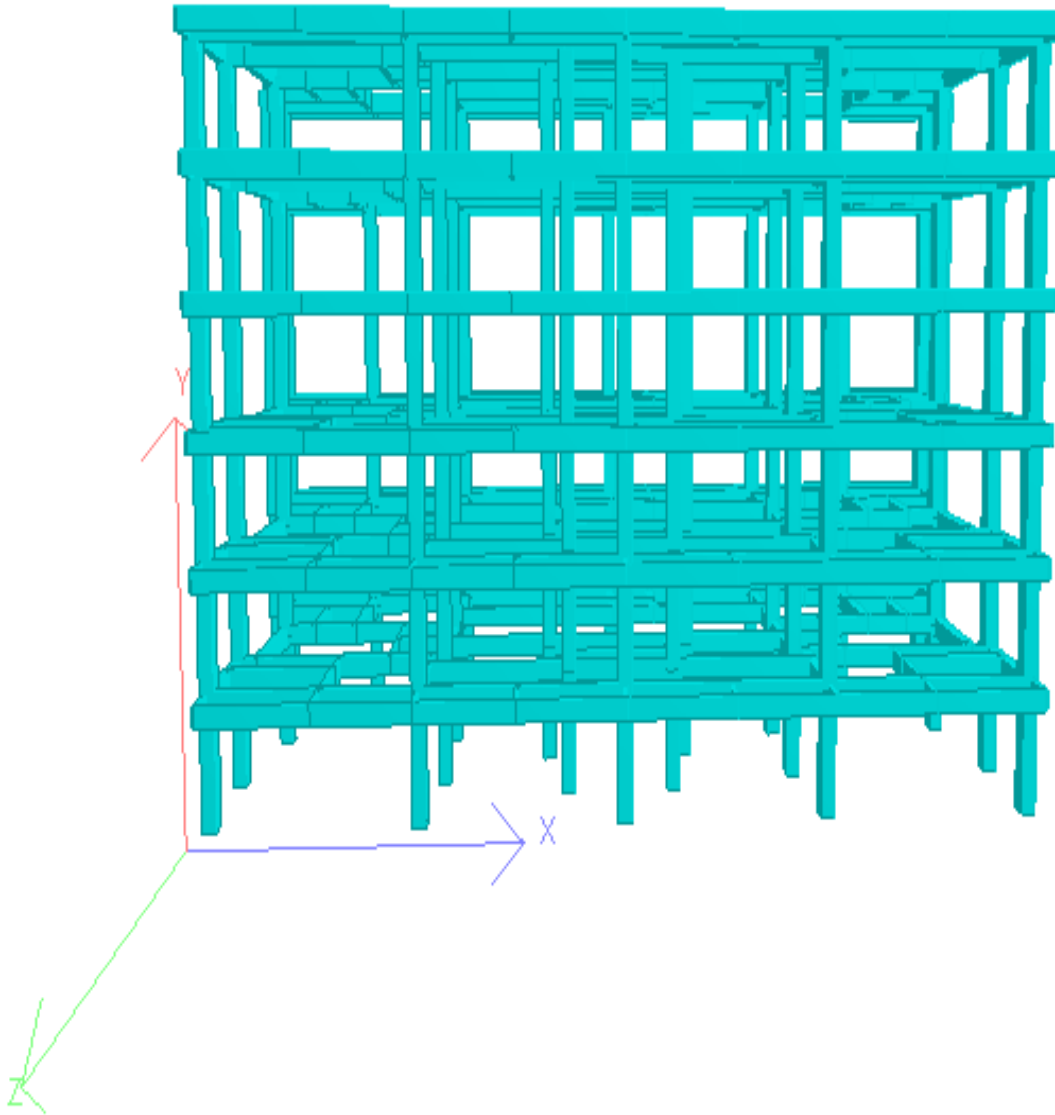


Figure 11: Whole building with member properties applied to all the members

- Member release: The secondary beams were released in Y and Z directions releasing  $M_y$  and  $M_z$  at appropriate points.

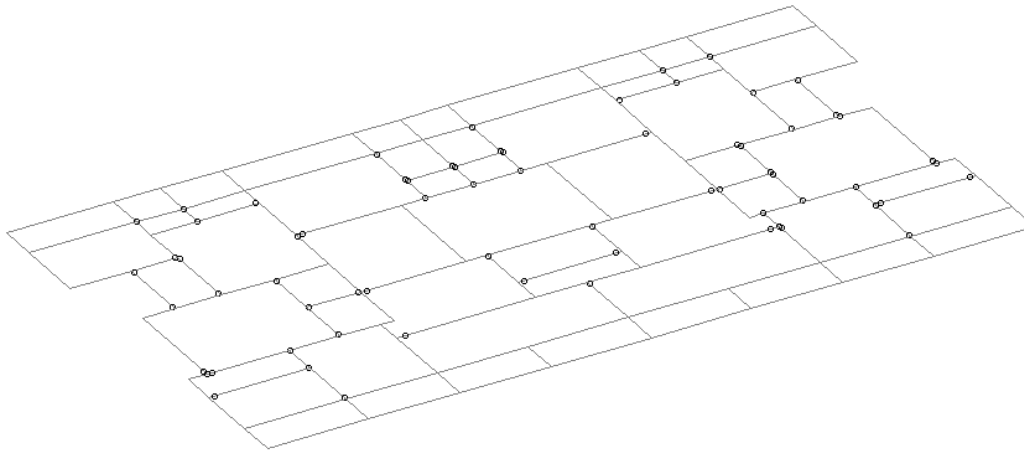


Figure 12: Showing member release of first floor beams

### 3.3 Loading:

- Dead load:

Includes self-weight of all members + Brick Load + Floor load from slabs

Brick load due to 2.4 m high brick wall and 250 mm thick and of  $2 \text{ T/m}^3$  density =

$$2.4 \times 0.25 \times 2 = 1.2T \text{ per } m \text{ udl}$$



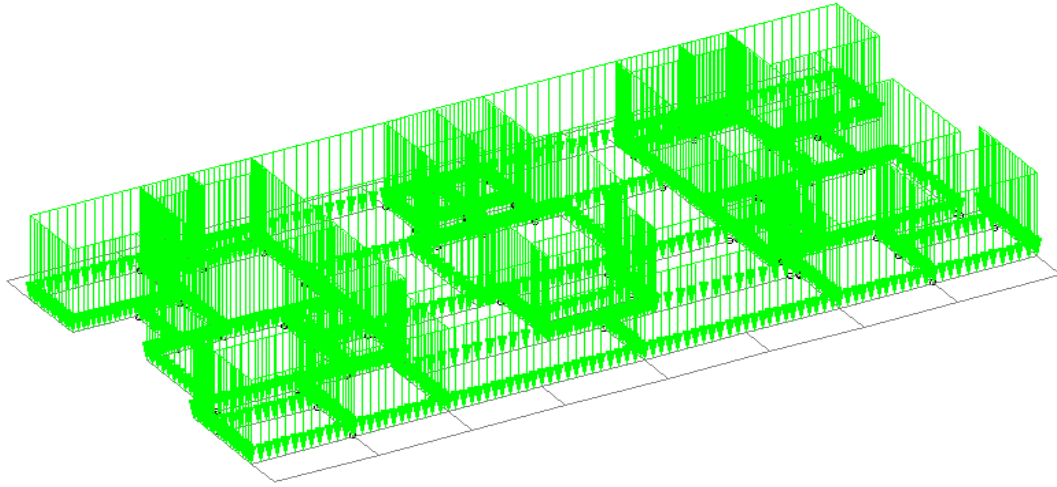


Figure 13: Showing application of brick load

- Floor load due to Slab:

Considering 150 mm thickness of slabs and  $2.5 \text{ T/m}^3$  density of concrete =

$$0.15 \times 2.5 \times 1 = 0.375 \text{ T per m}$$

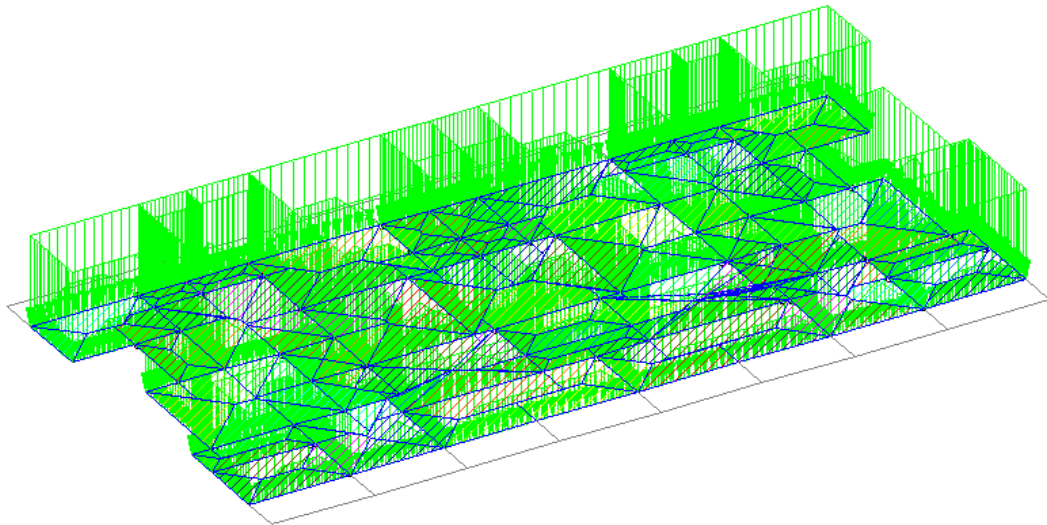


Figure 14: Showing application of dead load (brick load + slab load)

- Live load:

Taking maximum live load for residential building as per IS 875 (part 2) =  $0.3 \text{ T/m}^2$

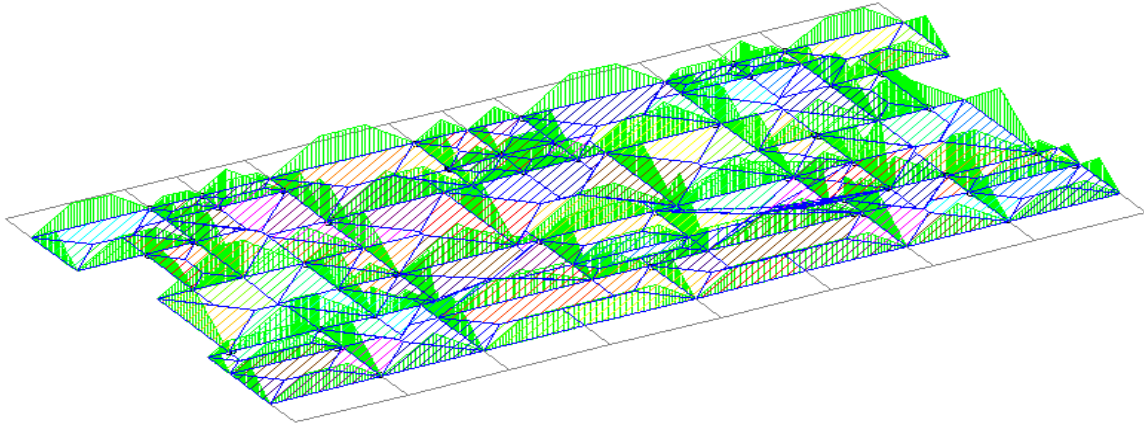


Figure 15: Showing application of live load

- Seismic load: The design seismic base shear was calculated as per IS 1893:2002 (part 1) for equivalent static procedure.

Table 5: Seismic weight calculation

Weight Calculation								
sl no.	category	no.	length (m)	breadth (m)	height (m)	volume (m3)	density (T/m3)	weight (T)
1	columns	28	0.5	0.4	3	0.6	2.5	42
2	beams	110	0.5	0.25*	0.5	0.0625	2.5	18
3	slab	-	24.55	11.4	0.15	~42.0	2.5	81
4	brick load	-	212.58	0.2	2.4	102.03	2	205
5	imposed load	taking 25% of total live load						26.27
sesimic wt for all floors except roof							=	372.27 Tonnes
seismic wt. for roof							=	222.50 Tonnes
total seismic weight							=	2083.85 Tonnes
0.25* is taken as an average								
~slab volume is reduced by bricks and effective value is 32.38								

Table 6: Calculation of base shear

calculation of base shear			
factors	formula	value	remarks
Z		0.36	zone V value from Table 2 of IS 1893:2002
I		1	residential building value from Table 6 of IS 1893
R		3	from Table 7 of IS 1893:2002
T	$0.075(h)^{0.75}$	0.655	h= 18 m from foundation
Sa/g	$Sa/g = 1.36/T$	2.075	
Ah	$Ah = Z * I * Sa / 2Rg$	0.125	Ah = design horizontal seismic coefficient
W		2083.85 T	Total Seismic wt. of Building
Vb	$Vb = Ah W$	259.44 T	Design Base Shear

Table 7: Distribution of base shear along vertical direction

Calculation of Seismic force per storey in X direction		V <sub>b</sub> per storey
Floor	$W_i h_i^2$	$W_i h_i^2 / \sum_1^n W_i h_i^2$ (in Tonnes)
1 <sup>st</sup>	13401.8	18.87
2 <sup>nd</sup>	30154.0	42.45
3 <sup>rd</sup>	53607.0	75.47
4 <sup>th</sup>	83761.0	117.93
Roof	72090.0	101.50

Table 8: Distribution of base shear per node in X and Z directions for each floor

Seismic forces per node (T)		
Floor	X direction (5 nodes)	Z direction (9 nodes)
1 <sup>st</sup>	3.77	2.10
2 <sup>nd</sup>	8.49	4.72
3 <sup>rd</sup>	15.09	8.39
4 <sup>th</sup>	23.59	13.10
Roof	20.30	11.28

- Load combinations applied

Table 9: Load combinations as per IS 1893:2002 (part 1)

<b>1.5 (DL+LL)</b>
<b>1.2(DL+LL±EL)</b>
<b>1.5(DL±EL)</b>
<b>0.9DL ± 1.5EL</b>

- After the loading is completed, the structure was analyzed in STAAD.
- Member force details were taken from it as input data for calculating demand to capacity ratios

### 3.4 Computation of dcr:

DCR is the demand to capacity ratio of members. From the dcr value of beams and columns, we can know if they would be safe under the applied load combinations or not. The simple philosophy used for checking the safety of the members of the building and the essence of this project is :

<b>dcr</b>	<b>status</b>
<b>&lt;1.0</b>	<b>safe</b>
<b>&gt;1.0</b>	<b>fail</b>

The dcr value is calculated for beams and columns in case of flexure and shear.

The demands are input from the Staad after analysis.

The capacity of beams can be computed using formulae for ultimate moment capacity of doubly reinforced beams (as all beams in this building are doubly reinforced) using following equations:

$$M_u = 0.36f_{ck}bx_u(d - 0.416x_u) + (f_{sc} - f_{cc})A_{sc}(d - d') \quad (3.1)$$

$f_{sc}, f_{cc}$  = stresses in compression steel and concrete respectively corresponding to strain:

$$e_{cs} = \frac{0.0035(x_u - d')}{x_u} \quad (3.2)$$

$d$  = effective depth

$d'$  = cover at top

$$x_u = \text{depth of neutral axis} = \frac{(0.87 f_y A_{st} - (f_{sc} - f_{cc}) A_{sc})}{0.36 f_{ck} b} \quad (3.3)$$

For columns, the moment demand due to biaxial bending under axial compression was checked using the  $P$ - $M_x$ - $M_y$  interaction surface generated according to IS 456:2000. The demand point has to be located in the  $P$ - $M_x$ - $M_y$  space and a straight line is drawn joining the demand point to the origin. The line (extended if needed) will intersect the interaction surface at the capacity point. The ratio of distance of demand point from the origin to the distance of capacity point from the origin is the dcr of the column.

For this purpose, I used a C program which worked on the above principle. The maximum axial force ( $P$ ) and biaxial bending moment values ( $M_x$ - $M_y$ ) obtained from Staad were input in the program through a text file and it gave the dcr as the output through another text file.

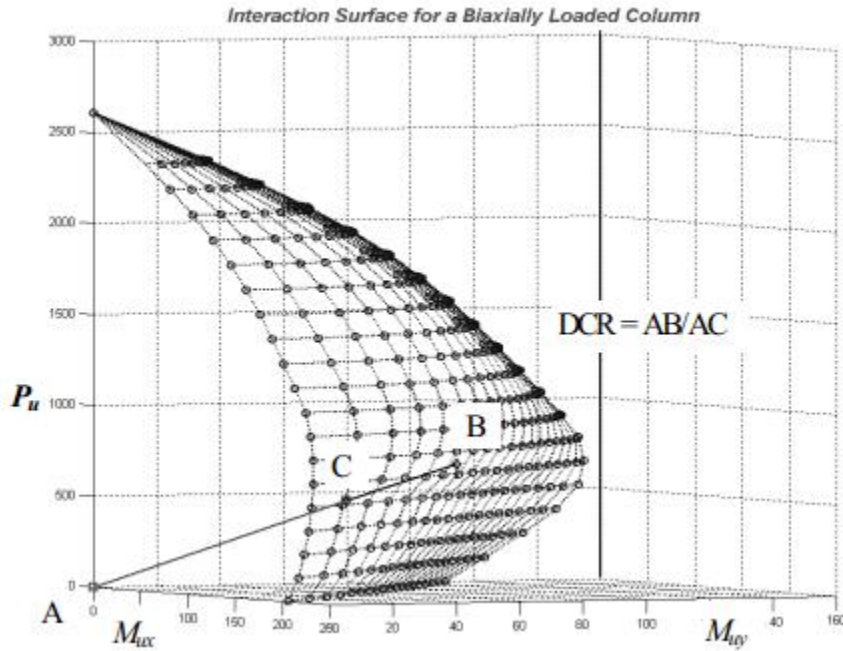


Figure 16: dcr of column under biaxial bending<sup>[10]</sup>

### 3.5 Results:

The results obtained for the dcr values and the status of the members in the building are discussed below. The results are for beams of level 2 and random columns were selected (specifically the foundation ones) and their dcr values were calculated.

Table 10: status of beams of level 2 in flexure

beams status in flexure			
beam no.	beam type	dcr	status
21101	1b3	2.08	fail
21102	1b3	1.99	fail
21103	1b3	1.66	fail
21104	1b3	1.60	fail
21105	1b3	1.59	fail
21106	1b3	1.57	fail
21107	1b3	1.74	fail
21108	1b3	1.87	fail
21111	1b6	0.01	safe
21112	1b6	0.01	safe
21121	1b4	-0.15	safe
21122	1b4	1.82	fail
21123	1b4	1.84	fail
21124	1b4	1.06	fail
21201	1b8	0.01	safe
21202	1b8	-0.97	safe
21203	1b8	-1.07	fail
21204	1b8	1.52	fail
21205	1b8	1.94	fail
21206	1b8	1.65	fail
21207	1b8	-0.26	safe
21208	1b8	-0.68	safe



<b>21209</b>	1b8	-0.70	safe
<b>21210</b>	1b8	0.01	safe
<b>21211</b>	1b6	0.01	safe
<b>21221</b>	1b5	0.01	safe
<b>21222</b>	1b5	1.87	fail
<b>21223</b>	1b5	2.43	fail
<b>21224</b>	1b5	1.67	fail
<b>21225</b>	1b5	2.46	fail
<b>21226</b>	1b5	1.87	fail
<b>21227</b>	1b5	0.01	safe
<b>21301</b>	1b9	2.45	fail
<b>21302</b>	1b9	1.17	fail
<b>21303</b>	1b9	-1.02	fail
<b>21304</b>	1b9	2.40	fail
<b>21305</b>	1b9	0.01	safe
<b>21306</b>	1b9	-0.69	safe
<b>21307</b>	1b9	-0.50	safe
<b>21308</b>	1b9	0.01	safe
<b>21311</b>	1b4	1.15	fail
<b>21312</b>	1b4	1.54	fail
<b>21313</b>	1b4	-0.74	safe
<b>21314</b>	1b4	-0.72	safe
<b>21315</b>	1b4	1.51	fail
<b>21316</b>	1b4	1.13	fail
<b>21401</b>	1b5	2.24	fail
<b>21402</b>	1b5	-0.12	safe

<b>21403</b>	1b5	-0.82	safe
<b>21404</b>	1b5	2.08	fail
<b>21411</b>	1b5	-0.08	safe
<b>21412</b>	1b5	-0.08	safe
<b>21421</b>	1b5	1.81	fail
<b>21422</b>	1b5	-0.88	safe
<b>21423</b>	1b5	-0.09	safe
<b>21424</b>	1b5	1.73	fail
<b>21501</b>	1b2	1.68	fail
<b>21502</b>	1b2	-0.38	safe
<b>21503</b>	1b2	1.42	fail
<b>21504</b>	1b2	1.57	fail
<b>21505</b>	1b2	1.55	fail
<b>21506</b>	1b2	1.57	fail
<b>21507</b>	1b2	1.55	fail
<b>21508</b>	1b2	1.41	fail
<b>21509</b>	1b2	-0.39	safe
<b>21510</b>	1b2	1.62	fail
<b>22101</b>	1b10	2.57	fail
<b>22111</b>	1b10	2.49	fail
<b>22112</b>	1b10	-2.42	fail
<b>22121</b>	1b10	-0.24	safe
<b>22132</b>	1b11	0.01	safe
<b>22141</b>	1b15	0.01	safe
<b>22142</b>	1b15	1.18	fail
<b>22143</b>	1b15	0.01	safe

22151	1b12	0.01	safe
22152	1b12	0.01	safe
22161	1b5	0.01	safe
22171	1b7	-0.98	safe
22172	1b7	-1.26	fail
22181	1b13	2.23	fail
22182	1b13	-1.11	fail
22201	1b5	2.76	fail
22202	1b5	-2.35	fail
22203	1b5	2.15	fail
22204	1b5	-1.11	fail
22205	1b5	-2.06	fail
22221	1b5	2.91	fail
22231	1b5	2.38	fail
22232	1b5	1.34	fail
22301	1b13	2.05	fail
22302	1b13	0.01	safe
22303	1b13	1.39	fail
22311	1b5	0.01	safe
22312	1b5	0.01	safe
22321	1b5	0.01	safe
22322	1b5	0.01	safe
22331	1b5	2.41	fail
22332	1b5	1.36	fail
22341	1b5	2.93	fail
22401	1b5	3.06	fail

<b>22402</b>	1b5	-2.33	fail
<b>22403</b>	1b5	2.44	fail
<b>22404</b>	1b5	-1.68	fail
<b>22405</b>	1b5	-2.11	fail
<b>22411</b>	1b13	1.14	fail
<b>22412</b>	1b13	0.01	safe
<b>22421</b>	1b7	0.01	safe
<b>22422</b>	1b7	0.01	safe
<b>22431</b>	1b5	0.01	safe
<b>22441</b>	1b12	0.01	safe
<b>22442</b>	1b12	0.01	safe
<b>22451</b>	1b15	-0.87	safe
<b>22452</b>	1b15	-1.00	safe
<b>22453</b>	1b15	0.01	safe
<b>22461</b>	1b11	0.01	safe
<b>22471</b>	1b10	-1.47	fail
<b>22501</b>	1b10	2.61	fail
<b>22502</b>	1b10	-2.37	fail
<b>22511</b>	1b10	2.59	fail

Table 11: status of beams of level 2 (first floor) in shear

beam status in shear			
beam no.	beam type	dcr	status
21101	1b3	0.88	safe
21102	1b3	0.97	safe
21103	1b3	0.97	safe
21104	1b3	0.88	safe
21105	1b3	0.88	safe
21106	1b3	0.96	safe
21107	1b3	0.97	safe
21108	1b3	0.87	safe
21111	1b6	0.50	safe
21112	1b6	0.53	safe
21121	1b4	0.68	safe
21122	1b4	1.81	fail
21123	1b4	1.83	fail
21124	1b4	0.68	safe
21201	1b8	0.00	safe
21202	1b8	0.94	safe
21203	1b8	1.53	fail
21204	1b8	1.89	fail
21205	1b8	0.00	safe
21206	1b8	1.43	fail
21207	1b8	1.89	fail
21208	1b8	1.53	fail

<b>21209</b>	1b8	0.94	safe
<b>21210</b>	1b8	0.00	safe
<b>21211</b>	1b6	0.56	safe
<b>21221</b>	1b5	1.49	fail
<b>21222</b>	1b5	0.77	safe
<b>21223</b>	1b5	0.00	safe
<b>21224</b>	1b5	0.77	safe
<b>21225</b>	1b5	0.00	safe
<b>21226</b>	1b5	0.77	safe
<b>21227</b>	1b5	1.49	fail
<b>21301</b>	1b9	2.46	fail
<b>21302</b>	1b9	1.78	fail
<b>21303</b>	1b9	1.48	fail
<b>21304</b>	1b9	1.48	fail
<b>21305</b>	1b9	1.48	fail
<b>21306</b>	1b9	1.48	fail
<b>21307</b>	1b9	1.78	fail
<b>21308</b>	1b9	2.46	fail
<b>21311</b>	1b4	0.90	safe
<b>21312</b>	1b4	0.00	safe
<b>21313</b>	1b4	1.99	fail
<b>21314</b>	1b4	2.01	fail
<b>21315</b>	1b4	0.00	safe
<b>21316</b>	1b4	0.90	safe
<b>21401</b>	1b5	1.31	fail
<b>21402</b>	1b5	1.78	fail

<b>21403</b>	1b5	1.78	fail
<b>21404</b>	1b5	1.31	fail
<b>21411</b>	1b5	1.69	fail
<b>21412</b>	1b5	1.70	fail
<b>21421</b>	1b5	1.72	fail
<b>21422</b>	1b5	1.31	fail
<b>21423</b>	1b5	1.31	fail
<b>21424</b>	1b5	1.72	fail
<b>21501</b>	1b2	1.30	fail
<b>21502</b>	1b2	2.95	fail
<b>21503</b>	1b2	2.25	fail
<b>21504</b>	1b2	1.06	fail
<b>21505</b>	1b2	2.90	fail
<b>21506</b>	1b2	2.92	fail
<b>21507</b>	1b2	1.06	fail
<b>21508</b>	1b2	2.24	fail
<b>21509</b>	1b2	2.95	fail
<b>21510</b>	1b2	1.29	fail
<b>22101</b>	1b10	1.68	fail
<b>22111</b>	1b10	1.24	fail
<b>22112</b>	1b10	2.20	fail
<b>22121</b>	1b10	0.69	safe
<b>22132</b>	1b11	0.54	safe
<b>22141</b>	1b15	1.34	fail
<b>22142</b>	1b15	2.17	fail
<b>22143</b>	1b15	0.00	safe

<b>22151</b>	1b12	0.95	safe
<b>22152</b>	1b12	1.68	fail
<b>22161</b>	1b5	0.00	safe
<b>22171</b>	1b7	0.65	safe
<b>22172</b>	1b7	0.66	safe
<b>22181</b>	1b13	1.16	fail
<b>22182</b>	1b13	0.00	safe
<b>22201</b>	1b5	1.42	fail
<b>22202</b>	1b5	1.45	fail
<b>22203</b>	1b5	1.42	fail
<b>22204</b>	1b5	1.26	fail
<b>22205</b>	1b5	0.00	safe
<b>22221</b>	1b5	1.06	fail
<b>22231</b>	1b5	2.59	fail
<b>22232</b>	1b5	1.52	fail
<b>22301</b>	1b13	1.16	fail
<b>22302</b>	1b13	2.35	fail
<b>22303</b>	1b13	1.50	fail
<b>22311</b>	1b5	2.33	fail
<b>22312</b>	1b5	1.49	fail
<b>22321</b>	1b5	2.33	fail
<b>22322</b>	1b5	1.49	fail
<b>22331</b>	1b5	2.59	fail
<b>22332</b>	1b5	1.52	fail
<b>22341</b>	1b5	1.07	fail
<b>22401</b>	1b5	1.42	fail



<b>22402</b>	1b5	1.45	fail
<b>22403</b>	1b5	1.42	fail
<b>22404</b>	1b5	1.26	fail
<b>22405</b>	1b5	0.00	safe
<b>22411</b>	1b13	1.16	fail
<b>22412</b>	1b13	0.00	safe
<b>22421</b>	1b7	0.65	safe
<b>22422</b>	1b7	0.66	safe
<b>22431</b>	1b5	0.00	safe
<b>22441</b>	1b12	0.95	safe
<b>22442</b>	1b12	1.68	fail
<b>22451</b>	1b15	1.34	fail
<b>22452</b>	1b15	2.17	fail
<b>22453</b>	1b15	0.00	safe
<b>22461</b>	1b11	0.54	safe
<b>22471</b>	1b10	0.69	safe
<b>22501</b>	1b10	1.28	fail
<b>22502</b>	1b10	2.20	fail
<b>22511</b>	1b10	1.74	fail

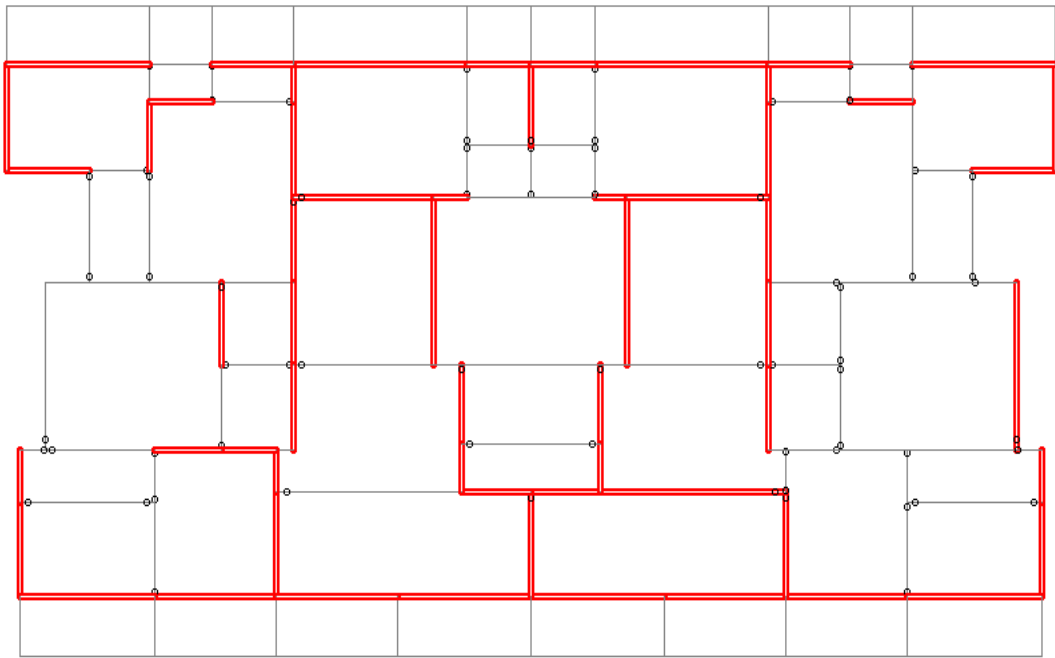


Figure 17: Beams failing under flexure

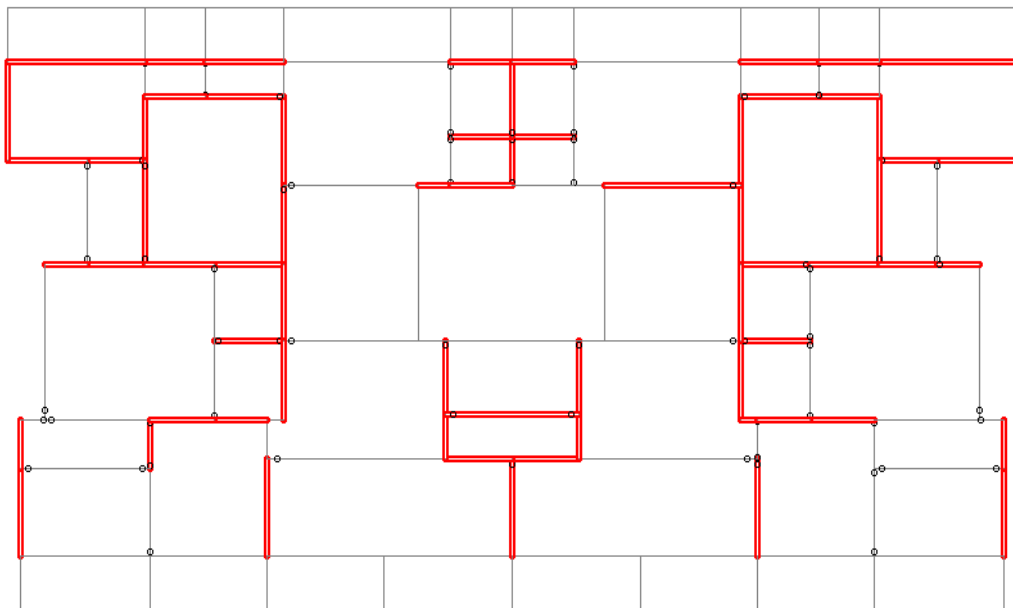


Figure 18: Beams failing under shear

Table 12: status of columns checked under biaxial bending

<b>columns under biaxial bending</b>			
<b>column no.</b>	<b>column type</b>	<b>dcr</b>	<b>status</b>
<b>3111</b>	nc2	2.10	fail
<b>1111</b>	nc1	2.92	fail
<b>1301</b>	nc3	2.56	fail
<b>1302</b>	nc3	2.32	fail
<b>2331</b>	nc5	2.35	fail
<b>4341</b>	nc4	2.15	fail
<b>4342</b>	nc4	2.13	fail
<b>5471</b>	nc2	2.12	fail
<b>5472</b>	nc2	1.29	fail
<b>1501</b>	nc1	2.89	fail
<b>1502</b>	nc1	1.93	fail
<b>7101</b>	nc2	1.80	fail
<b>7102</b>	nc2	1.27	fail
<b>7103</b>	nc2	1.24	fail
<b>3201</b>	nc5	1.97	fail
<b>3202</b>	nc5	1.84	fail
<b>3203</b>	nc5	1.76	fail
<b>6224</b>	nc5	0.91	safe
<b>6225</b>	nc5	0.78	safe

#### **4. Local retrofitting methods**

From the results obtained above for this building, it is clear that the members will fail under the applied load combinations as per IS 1893:2002 (part 1) and we have to provide retrofitting in the building. The scope of my study is limited to local retrofitting measures.

##### **4.1 Retrofit of columns:**

Retrofit methods of columns include:

- Concrete jacketing
- Steel jacketing
- Fibre reinforced polymer wrapping

The columns in this structure can be retrofitted by concrete jacketing, which is the most popular method of seismic retrofit in columns. There are two main purposes of jacketing of columns:

- It increases the shear capacity of columns
- It improves the flexural strength of columns by the longitudinal steel of the jacket made continuous through the slab system and anchored with the foundation.

It is achieved by passing the new longitudinal reinforcement through holes drilled in the slab and by placing new concrete in the beam column joints

The method is straightforward and increases both strength and ductility. But the composite deformation of the existing and the new concrete requires adequate dowelling to the existing

column. The mix design of the new concrete, surface preparation of the existing column and the choice of bonding material are also important.

The disadvantages of concrete jacketing are:

- Drilling of holes
- Increase in size of the column
- Placement of ties at the beam-column joint

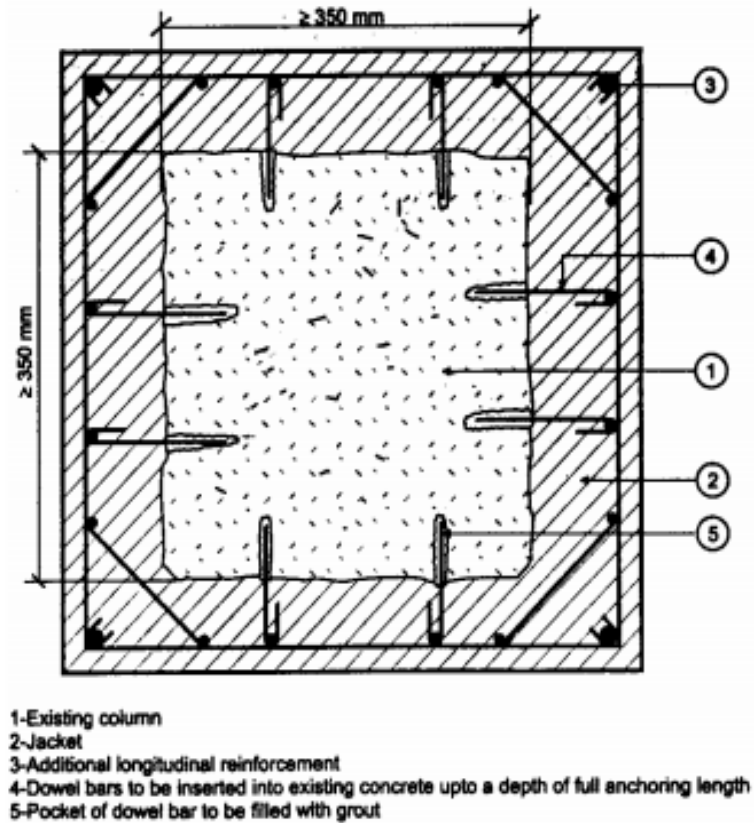


Figure 19: Concrete jacketing<sup>[10]</sup>

#### 4.2 Retrofit of beams:

Beams can be strengthened by:

- Concrete jacketing
- Steel plating
- Use of FRP bars
- External prestressing

In this structure we can use concrete jacketing as well as steel plating. The scope of my study is limited to retrofitting of beams by steel plating. In steel plating, steel plates are glued to beams to improve their flexural and shear capacities. It increases the strength and stiffness of the beams and reduces the crack width<sup>[10]</sup>.

Advantages of steel plating:

- Addition of steel plates is simple and can be rapidly applied
- Does not reduce the storey clear height significantly
- Can be applied while the building is still in use
- Relatively small increase in size of the existing section

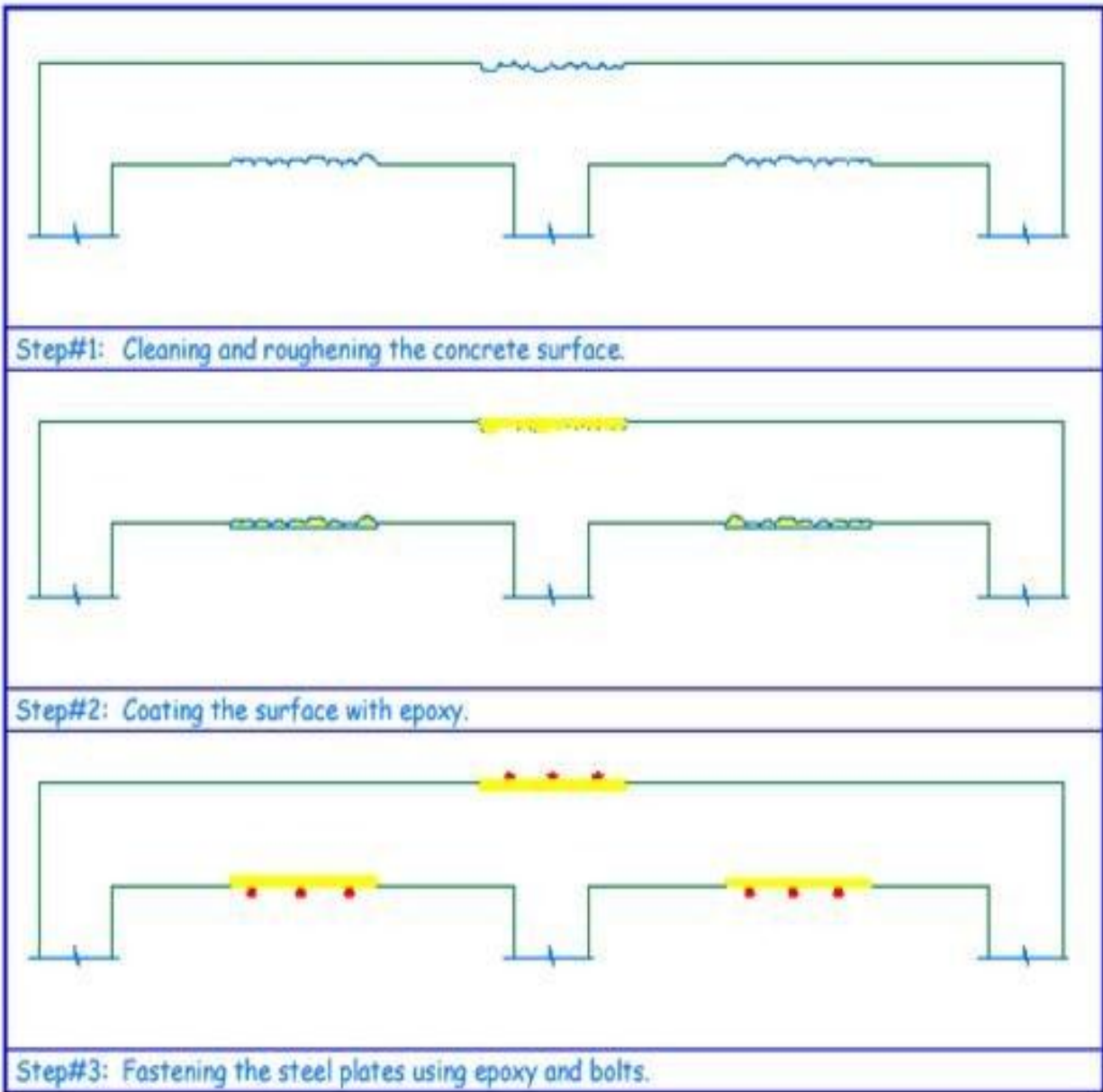


Figure 20: Showing steps of steel plating<sup>[11]</sup>

**Steel plating:**

Sample Calculation:

Beam no: 21311

Size : 200 X 500 mm

Type: 1b4

Original capacity = 272.43 KNm

Target Capacity = 315 KNm

Steel plate of thickness 2mm added to both tension and compression face

$d_p = 2 \text{ mm}$

$d = \text{effective depth of beam} = 500 - d_c (40 \text{ mm}) = 460 \text{ mm}$

$f_{pc} = f_{pt} = \text{strees in steel plate in compression and tension corresponding to strain } e_{cs}$

$e_{cs} = 0.0023$  (calculated while calculating neutral axis)

$f_{pc} = f_{pt} = 340 \text{ N/mm}^2$  for Fe 415

providing width of steel plate =  $b = \text{width of beam} - 2(50 \text{ mm side cover}) = 100 \text{ mm}$



Strength added by steel plating =

compression side + tension side

$$\text{compression side} = f_{pc} \cdot A_{pc} \left( \frac{d_p}{2} + d \right) \quad (4.1)$$

$$\text{tension side} = f_{pt} \cdot A_{pt} \left( \frac{d_p}{2} + d_c \right) \quad (4.2)$$

$$= 340 (4 * 100)(1 + 460) = 62.7 \text{ KNm} + 340(4 * 100)(1 + 40) = 5.57 \text{ KNm}$$

$$= 68.27 \text{ KNm}$$

Therefore total capacity of beam after steel plating

$$= 272.43 + 64.24 \text{ KNm} = 336.67 \text{ KNm}$$

Required capacity = 315 KNm

Hence target capacity achieved by steel plating

Similarly steel plating can be done for the following beams. Thickness of beam can be increased by 1 or 2 mm (from 4mm) to achieve the desired capacity.

Table 13: beams of first floor on which steel plating can be done

eligible for steel plating	
beams	dcr
21124	1.06
21203	-1.07
21204	1.52
21302	1.17
21303	-1.02
21311	1.15
21312	1.54
21315	1.51
21316	1.13
21503	1.42
21504	1.57
21505	1.55
21506	1.57
21507	1.55
21508	1.41
22142	1.18
22172	-1.26
22182	-1.11
22204	-1.11
22232	1.34
22303	1.39
22332	1.36
22404	-1.68
22411	1.14

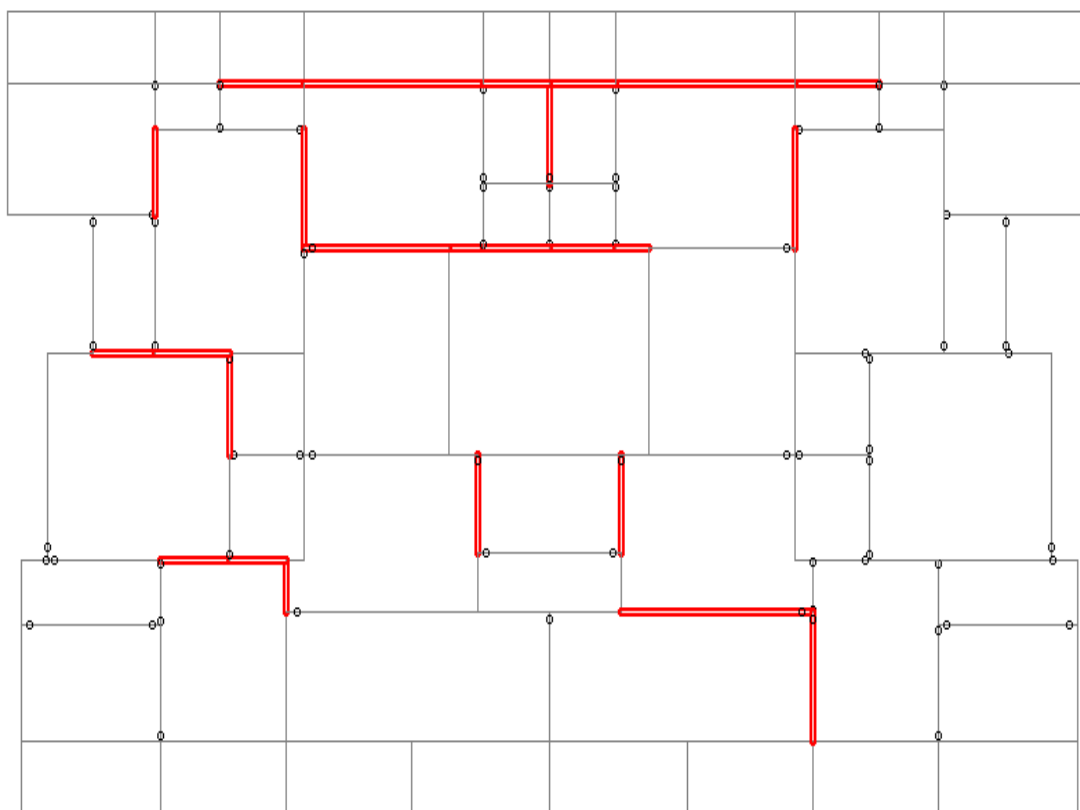


Figure 21: showing beams of 1<sup>st</sup> floor eligible for steel plating

### 5.1 Summary:

The purpose of this project was to assess the seismic vulnerability of an existing RC structure and to provide for retrofit in case the members fail. The building under study in this project was an existing multi-storeyed residential building in Guwahati. The plan and reinforcement details of the building were provided. I modeled the building in STAAD Pro software and applied seismic load combinations to it. Equivalent static procedure as per Indian Standard IS 1893:2002 (Part 1) was used to compute the seismic forces. The members' adequacy was assessed by computation of their dcr (demand to capacity ratio) values. The demand of individual members was obtained after analysis from STAAD Pro software and the capacity for the corresponding members was calculated, the ratio of the two gave the dcr values. The simple concept that if the dcr of any member is greater than one would result in the failure of that member under the applied loads was used to find out the status of the members under flexure and shear.

### 5.2 Conclusion:

The results for first floor beams and a large sample of columns showed that a number of beams and all the foundation columns checked were found to be deficient under the applied seismic load combinations. Number of beams failing under flexure was more than the number of beams failing under shear. The dcr of columns under biaxial bending gradually decreased with height, although it was greater than one in most of the cases.

For providing retrofit measures for the deficient members, concrete jacketing was found to be a suitable method for retrofitting of columns. It was also concluded that steel plating would be an efficient method of retrofitting of a number of deficient beams.

## 6. References

- [1] [http://help.solidworks.com/2012/English/SolidWorks/cworks/c\\_Definitions\\_Response\\_Spectra.htm](http://help.solidworks.com/2012/English/SolidWorks/cworks/c_Definitions_Response_Spectra.htm)
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